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STRESS MEASUREMENTS ON THE SAN LEANDRO CREEK BRIDGE

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Introduction

Comparison of the actual stresses developed in highway structures with the design stresses which are obtained by standard methods of analysis is a subject which has long interested structural engineers. Recent advances in the science of experimental stress analysis have greatly simplified the problem of making the large scale field investigations which are required to determine the actual stresses in full scale structures. Thus, in order to study methods and develop techniques for performing such large scale investigations, the Institute of Traffic and Transportation Engineering of the University of California in the spring of 1950 initiated an extensive program of strain and deflection measurements in a typical slab and girder highway bridge, in cooperation with the U.S. Bureau of Public Roads and the California Division of Highways.

In addition to the basic purpose of developing field stress measurement techniques, specific objectives of the test program were:

(1) to study the manner in which loads applied to the slab were distributed to the main girders,

(2) to evaluate the degree of composite action which existed in a structure in which no special anchoring devices were provided to develop bond between the concrete deck and the steel girders, and

(3) to obtain data from which the so-called impact stresses, i.e. the increase of stress caused by the motion of the vehicle, might be evaluated.

It is the purpose of this paper to discuss briefly the phase of the research program devoted to the mechanics of obtaining the required stress information from the structure. The presentation is divided into two parts, first will be described the design and installation of the instrumentation used in obtaining the basic records, then the methods of reducing these records and some of the difficulties encountered in obtaining reliable data will be discussed. It must be pointed out here that analyses and interpretation of the stress information

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1. A paper to be presented at the 1953 Spring Convention of the American Society of Civil Engineers in San Francisco, California
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is not a part of this paper. However, the complete report of the entire investigation is available as Research Report No. 11 of the Institute of Traffic and Transportation Engineering, University of California.

Instrumentation

The design and installation of the instrumentation used in obtaining the basic records for this project may be described most effectively by means of figures, so we shall proceed directly to the first group of slides.

Description of the Test Structure

Figure 1 shows an aerial view of the structure selected for the test program. It is the San Leandro Creek Bridge, a link in the Eastshore Freeway, located at the southern limits of the city of Oakland, California. The bridge consists of two parallel, 2-lane structures of 23 spans each. Each structure has an 8 in. concrete deck supported by three longitudinal girders on 11 ft. centers. Every third span has a suspended section, hinge supported on the cantilevered ends of the girders which are continuous over the adjacent two spans. Two representative spans, spans 19 and 20 of the east structure, were selected for testing; 19 being a typical suspended span and 20 a typical continuous span.

Figure 2 shows the layout of the test spans. The outer girders are supported directly on the columns and the middle girder is supported by transverse beams framing into the outer girders. Diaphragms are provided in each span to aid in distributing the load among the girders.

Design of the Instrumentation

The manner in which load is distributed to the girders by the slab and diaphragms is indicated by the relative magnitudes of the bending moments acting in the three girders at a given cross-section. For the purpose of studying distribution of load, main gage stations were established at four sections in the test spans, as indicated in Figure 2. In addition, supplementary stations were established at Station 19.7 and 20.3.

Figure 3 shows the manner in which strain gages were located at the midspan gage stations. Carlson strain meters were located at two levels in the concrete, and SR-4 strain gages were applied to the flanges of the girder. Knowing the properties of the section, it is possible to compute the total moment carried by each composite girder from the strain measurements provided by these gages. For the purpose of these computations, the deck was assumed divided into three longitudinal strips, each strip being assumed to act with the girder which supported it.

Figure 4 shows a detail of the arrangement of strain gages at a station adjacent to a girder support. At these stations, additional gages (rosettes) were provided in the web of the girder, from which shear forces and a check on the longitudinal strains could be obtained. Fig. 4 also shows how the degree of composite action existing at a given gage station may be evaluated by plotting the strains indicated at the several gage positions across the depth of the girder. If the concrete is effectively bonded to the steel, the composite section will act as a unit and strains will vary linearly through the entire depth (Fig. 4b). On the other hand, if bond is completely lacking the concrete will bend independently of the steel, and the strain diagram will show a discontinuity as in Fig. 4c. In addition to these main girder gage stations, other

stations were provided to measure forces in the hinges and in the columns, and moments in the transverse beams and diaphragms.

Installation of Gages

The principal strain measuring device used in this project was the SR-4 resistance wire strain gage. This gage consists of a very fine wire sandwiched between pieces of paper so that it may be bonded to the structure. As the structure is strained, the fine wire is strained equally causing its resistance to change by an amount proportional to the strain. Two types of SR-4 gages were used on the San Leandro Creek Bridge, the A-1 for normal strain measurements and the AR-1 rectangular rosette for measuring both normal and shear strains. The installation of the gage followed the usual laboratory procedure in most respects. The metal was smoothed and cleaned with a nitro-cellulose cement.

The most interesting part of the gage application routine was getting into position to do the work.

Figure 5 shows one of the technicians applying gages to the right girder. The lead wires which appear in the picture have already been connected to gages further down the structure. The steel framing in the test spans shows up clearly here: main girders, transverse beam, diaphragms and hinge details at the end of the suspended span.

Figure 6 illustrates a typical installation of a rosette and an ordinary SR-4 gage, before the lead wires were attached.

Figure 7 shows a similar installation after the lead wires were connected and water-proofing applied. Waterproofing of the gages was accomplished by coating them thoroughly with an asphaltic compound which had been heated sufficiently to render it quite fluid. The effectiveness of this waterproofing technique is demonstrated by the fact that none of the gages on the structural steel was grounded out at any time during the test program, in spite of the fact that the structure was thoroughly hosed down after the deck was poured in order to wash off the cement which had dripped through the forms.

The temperature compensating or "dummy" gages are also visible in this figure. These consisted of additional SR-4 gages similar to those applied to the structure but mounted on separate pieces of steel and fastened to the girder adjacent to the active gages. The circuit used in measuring strains with the SR-4 gages is so designed that stress-free deformations in the structure due to thermal expansion are balanced by the equal thermal strains developed in the dummy gages. Thus, only strains due to stress (which will not affect the dummy gages) are indicated by the measuring circuit. Separate compensating gages were used in every SR-4 gage circuit installed in the structure, in order that the temperature compensation be as complete as possible.

In addition to those on the girders, SR-4 gages were applied at 28 locations on the reinforcing steel in the deck slab.

Figure 8 shows a typical installation of the SR-4 gages on the reinforcing bars. The procedure followed in applying these gages had been used with success in previous laboratory tests. The active gages were coated with two layers of asphaltic compound, the dummy gages were coated with the same material and sealed in brass tubes. In spite of these precautions, however, very low resistances to ground were measured in many of the circuits, indicating the failure of the waterproofing. For this reason, these gages were considered unreliable and were not used in the test program. The reason for the failure

of the waterproofing technique is not known, but it is believed that the weak zone in the protective coating was at the point where the individual lead wires extended beyond the cable shield and were soldered to the gage.

Strains in the concrete deck were measured with the Carlson strain meter, a resistance wire gage operating on essentially the same principle as the SR-4 gage.

Figure 9 illustrates Carlson strain meters in three stages of assembly. The outer protective shell has been removed from the lowest gage in order to show its internal construction. Strains in the concrete into which the meter is cast cause its end flanges to move relative to each other, thus developing strains in the resistance wire coils. The flexible brass protective tube is shown in place on the middle gage. The upper gage has been covered with the cloth sleeve which serves to keep the concrete from bending to the protective shell and thus permits the flanges to strain freely.

The Carlson gages were installed by fastening them in the proper position, either to the reinforcing steel or to the girders, and then packing the concrete carefully about them so as to avoid disturbing their position.

Figure 10 shows two Carlson gages in position with the concrete about to be placed around them.

Deflections of the bridge were measured with variable inductance gages. This type of gage consists of a coil of fine wire mounted in a base which is firmly fastened to the ground, and a steel slug attached to the end of a pipe suspended from the lower flange of the girder. As the girder deflects, the steel slug is caused to move up or down in the coil, thus causing a change in its inductance. By calibration, this change of inductance can be related to the deflection of the girder. A typical deflectometer installation is visible in Figure 12.

In concluding the discussion of the gage installation program, the following statistics are offered. A total of 220 active SR-4 gages and 184 dummy gages were used throughout the structure requiring 129 separate ciruuits for the SR-4 gage measurements. In addition, 16 Carlson strain meters and eight deflectometers were installed. A separate circuit was required in each of these gages making a total of 153 circuits required for all measurements.

Recording Equipment

All gages were connected to the recording and indicating equipment by means of three lead, shielded microphone cables having a waterproof outer cover. The lead wires were supported on the lower flanges of the girders and were brought down to an instrument shack located under the structure.

Figure 11 shows the interior of the instrument shack with the lead wires entering through the opening at the upper rear, and connecting to three banks of switches. The switches for the SR-4 gage circuits are contained in the large box at the left, the Carlson gage switches are seen at the right center, and the deflectometer switch box is mounted on the rear wall. Static readings of the various gages were made by connecting standard SR-4, Carlson or deflectometer static indicating instruments to the corresponding switch boxes and switching the gages into the circuit one at a time.

For dynamic recordings of strains and deflection, additional leads were connected from the switch boxes to a trailer housing the synamic recording equipment.

Figure 12 presents an exterior view of the instrument trailer. (Note also the deflectrometer installations visible in this figure)

Figure 13 shows an interior view of the instrument trailer in operation. Dynamic recording equipment used in this test program included two 12 channel strain gage control and amplifier units, one 12 channel gage control unit without amplifiers, and three 24 channel oscillographs. With this equipment it was possible to record simultaneously the output of the eight deflectometers plus only 24 SR-4 or Carlson gage circuits. Thus in order to obtain dynamic records from all of the strain gages, it was necessary to repeat the load runs several times with different combinations of strain gages connected to the oscillographs.

Test Vehicle

Figure 14 shows the vehicle used for loading the bridge in most of the test work. It is a standard Euclid earth moving truck loaded with sand and steel ingots to a gross weight of 67,000 pounds. Of the total load, approximately 50,000 pounds was carried by the rear axle and 17,000 pounds by the front axle.

During the moving load tests, coordination between the truck driver and the oscillograph operators was provided by the test engineer shown on the bridge deck using the field telephone set.

Control Tests

Concurrently with the instrumentation of the structure, a program of control tests was carried out to determine the properties of the materials used in the structure. The most important property, from the standpoint of the stress measurement program, was the modulus of elasticity of each of the various materials. The modulus of the steel was obtained from standard tensile tests performed on specimens taken from flanges and webs of the girders and from the reinforcing bars.

Determination of the concrete modulus of elasticity was a more complex problem because its modulus varies with the weather conditions. Thus it was necessary to devise specimens which would simulate as closely as possible conditions within the test slab, and to test these specimens continuously during the stress measurement program. Two types of specimens were used: standard 6 in. x 12 in. cylinders used in compression tests, and 10 feet long by 10 inches wide unreinforced rectangular beams used in flexure tests. All beams were sealed on the sides and exposed to the air at the top surface. Three of the beams were made 8 in. deep and had their bottom surface exposed to the air, to simulate conditions in the slab between the girders; while the other three were made 10 in. deep and were sealed on the bottom to simulate conditions over the girders. They were stored on the deck of the bridge so as to be exposed to the same temperature and moisture conditions as the slab. Carlson strain meters were embedded in the beams at mid span and were used to measure the flexural strains.

Figure 15 shows a form for an 8 in. beam with the Carlson gages fastened in place ready for pouring the concrete.

Figure 16 shows the procedure used for testing the beams in flexure. Loads were applied simultaneously at the one-third span points by two hydraulic jacks connected into a single pressure system. The jacks exerted upward

forces against the test beam and downward forces against a wooden beam suspended below the test beam. Thus the test load opposed the force of gravity making it possible to develop sizeable strains without overstressing the unreinforced concrete.

Figure 17 presents the results of the moduli tests on the concrete. The very sizeable influence of moisture conditions upon the modulus is quite apparent here. As the beams dry out, the modulus decreases steadily; however, when rains saturate the concrete, the modulus increases again. Because of their smaller size and greater degree of exposure to the atmosphere, the 8 in. beams are affected more strongly by weather conditions than are the 10 in. beams, but the same tendencies are apparent in both cases. The compression test specimens exhibit similar trends, but since they were stored in the laboratory, they were affected only by changes in humidity and thus did not show as marked changes as did the beams stored in the field.

Figure 18 shows that shrinkage of the concrete also is affected to a marked degree by weather conditions. It is interesting to note that during the protracted period of rainfall the concrete absorbed enough water to expand again almost to its initial size.

Scope of Test Program

The primary effect under investigation was the behaviour of the structure under applied live loads, both static and dynamic although certain other auxiliary measurements were made. It was intended initially that most of the information necessary for the determination of these live load effects would be obtained by static readings of the gages for various longitudinal and transverse positionings of the test vehicle. This rather complete survey of the static effects was to have been supplemented by certain test runs with rapidly moving test vehicles at various speeds to evalute the dynamic effect.

In the course of making the initial series of static test it was discovered that, in spite of extensive precautions to eliminate sources of error, there was a very poor degree of repetition of results for duplicate tests and a general failure of gages to return to a stable no load reading when the test vehicle was removed from the structure. The next section of this paper will present the investigations by which the reasons for these discrepancies were isolated and the resulting modifications of the test program which led to a procedure for obtaining reliable test data.

Investigation of Sources of Error

As will be recalled from the previous discussion, all of the precautions normally taken in the laboratory were employed in the installation of the measurement devices in the field. Therefore, when difficulty was encountered in obtaining results comparable in accuracy and consistency with those normally obtained in laboratory, investigations were made of possible new sources of error peculiar to a field test setup. The difficulties were finally traced to three effects:

(1) Inadequate temperature compensation due to rapid rise or fall of ambient temperature, and difference in heating and cooling rates for structure and "dummy" gages.
(2) Actual stresses in the structure due to causes other than load, especially those due to temperature.

(3) Actual stresses in the structure due to highly localized effects in the vicinity of the load which were very sensitive to the positioning of the load.

The effectiveness of the temperature compensation at certain locations subject to rapid heating and cooling was checked by obtaining temperatures of the structural steel, the concrete slab and the dummy gage strips by the use of resistance thermometers and thermocoples. It was found by means of these measurements that, during the morning and later afternoon hours when temperature changes of the surroundings were rapid, there was at times a difference of one or two degrees in the temperatures of the "dummy" gage and the structure at the location of the corresponding active gage. This in itself would account for an apparent strain of about 10 to 12 microinches. This effect was particularly troublesome on sunny days, whereas on cloudy days it was hardly noticeable.

The second source of trouble proved to be the most difficult to cope with. A series of "no load" readings taken during the course of a single day without any load on the structure indicated changes in the strain at some of the gage locations very much in excess of what could be accounted for by inadequate temperature compensation. In fact, some of these differences were even larger than those caused by the application of the live load. A continuous record of temperatures at various points on the structure during this same period pointed to the reason for this difficutly. A correlation of the records of no load strains and temperatures shows that large and rapidly varying actual stresses are set up in the structure due to the changes in the ambient conditions. While these stresses are probably not important from the design standpoint, they certainly must be considered when conducting field stress measurements. These stresses are of several types as follows:

(1) The temperature stresses of the type inherent in any indeterminant structure. For instance, in this structure where we have a hinge joint between the girder and the supporting columns, we get first of all a bending of the columns and some direct stress in the girders due to the changes of girder length with temperature. In addition, if the slab heats faster than the girders, we get a tendency for the two span continuous beam to hump up and lift off the center support, causing a redistribution of the dead load reactions, with subsequent stresses.

(2) Warpage stresses due to non-symmetrical heating. In this structure a rapid heating of the lower flange of one of the outside girders, such as produced by early morning sun, causes this girder to tend to bend downward. It is restrained by the slab and also by the adjacent unheated girders, setting up a further complexity of stresses.

(3) Internal stresses Even if only a single unrestrained member is considered, we will have actual stresses due to temperature changes unless the temperature gradient across the cross section is linear. Any non-linearity of temperatures will produce direct stresses to achieve equilibrium and strain compatibility in the section.

The stresses produced by all of these temperature effects are quite difficult to evaluate or measure. Furthermore, as stated above, they are not constant, but are continuously varying. The successful elimination of the errors of live load strain due to this "background ' effect of temperature effects requires that the no load reading, loaded reading, and check on the no load reading be made during a time interval short enough that no appreciable changes

occur in these background effects during this interval. By actual test it was found that significant changes might occur in intervals greater than 15 minutes. This imposes a serious limitation on the number of measurements which can be made with a single loading of the structure, for with two experienced operators using the indicating equipment, it was found that 15 gages were all that could be read within this time. In a test program as extensive as this one, in which up to 150 circuits were read for each lateral and longitudinal position of the test load, this would require an unreasonable number of repetitions of the positioning of the live load.

This repositioning of the live load is in itself responsible for the third source of error. In some of the gages local strains were produced which were extremely sensitive to small difference in positioning of the load. This problem will be discussed later, after explaining the use of the dynamic recording equipment.

Interpretation of Dynamic Records

A typical oscillographic record is reproduced in Fig. 19, together with a portion of the calibration records for this series of runs and necessary identifying information. The procedure for producing such a record begins with the connection of the gages selected to the various channels of the oscillograph and introducing electrical elements of resistance and capacitance into this circuit to bring the reading of each channel to the zero position. This operation is accomplished by visual obervation of the light spots on the galvanometer scale, after which a photographic record is made by running off an inch or so of the sensitized paper. This operation produces the row of short "zero" marks along the left edge of the record. The vehicle is now brought up to proper speed and run across the test spans in the selected lateral position, which in this case was close to the right hand curb. At the same time the sensitized paper is moved at the predetermined speed past the recording aperture of the galvanometers, obtaining the continuous record of strain in each gage circuit. After the run, a final set of "zeros" which can be seen near the right hand edge of the record was obtained. The total time elapsed between initial and final zeros was in most cases less than one minute, so that strain variations due to temperature changes are extremely small. That they would be appreciable over longer periods of time can be seen by noting the initial zeros for the next run, particularly channel No. 11, which shows a decided shift. The interpretation of this record for any point on a given channel "trace" requires making two measurements (1) the value of the strain and (2) the position of the load. The first of these is represented by the excursion of the trace at the point in question from the straight line connecting the initial and final zeros. For example, at point A on the trace of channel 2, this excursion is 0.74 inches on the original record. In order to relate this measurement to strain in microinches per inch at the gage point, it is necessary to make a calibration for each series of test runs. A portion of the record of such a calibration is reproduced in the lower left hand side of the figure. A known amount of resistance is introduced into the circuit for each gage channel in turn, causing a sharp displacement of the trace, as at point B. By the known electrical properties of the circuit, this can be converted by calculation to an equivalent strain in the gage. In this case, the resistance introduced was equivalent to a strain at the gage of 64.6 microinches per inch. The trace at B in calibration measured 0.367 inches on the original, hence we have a calibration factor of 176 microinches per inch per inch of trace

deflection. The strain at point A in the test run is therefore 0.74 x 176 or 130 microinches per inch.

The position of the vehicle at the instant at which these strains existed is found by reference to the vehicle position trace between strain traces 6 and 7. This circuit was connected to traffic counting type air switches actuated by the wheels passing over hoses on the deck at stations 19.0, 20.0, and 21.0. Measurements from the "pips"made by the wheels permit determination of vehicle position and speed.

The examination of these oscillograph records made clear the reason for the erratic behavior which had been noted in some of the slab and top flange girder gages. Trace No. 11 of this record is for a Carlson Gage in the slab over the center of the right girder at station 20.5, a section of normally positive moment. It will be noted that the trace is distinguished by two sharp "pips" in the record at point (C). This gage shows a normal buildup of compression as the load approaches the gage point, but these two sharp pips which indicate a sudden increase in compression occur as the front and rear wheels pass over the gage location. Inspection of the trace for the bottom gage in the slab (No. 10) at this point shows a similar sudden reduction of compression for the corresponding instant, as does also the trace for each of the gages attached to the adjacent top flange of the girder (Traces 1 and 4). The explanation of this effect is found by a consideration of the localized transverse bending stresses produced in the slab. As the wheels of each axle straddle the gage location, strong transverse bending strains are developed in the slab, tensile on top and compressive at the bottom (see Fig. 20). Due to Poisson's effect, these transverse strains produce localized longitudinal strains in the slab and top flange of the girder, compressive on top and tensile at the bottom. It is the superposition of these localized strains upon the longitudinal bending strain of the composite girder which produces the "pips" in the records. The localized effects were eliminated in evaluating the records by sketching a smooth curve cutting off the "pip" from the oscillograph trace.

It will be noted that this strain is measured to the smooth line which can be drawn through the actual trace, thus yielding a value free of oscillations due to the vibration of the span at its own natural frequency. This value can be shown theoretically to differ by a small amount from the strain which would be produced by a static test with the load placed in the same position. The difference between the two is a function of the speed of the vehicle as related to the natural frequency of vibration of the span. For very slow speeds, such as this 5 mph test run, the dynamic and static strains are nearly identical, differing theoretically by less than one per cent. Thus it is permissible to replace the static tests by low speed dynamic runs.

Use of the dynamic recording equipment in place of the static indicators has many advantages in large scale field investigations such as this. First, it eliminates the many difficulties associated with temperature changes, since each test can be completed before appreciable changes in temperature occur; second, it saves a large amount of time in obtaining the test data; and third, it gives a more complete picture of the strains within the structure, thus making possible the detection and elimination of local effects.

Therefore, it was decided to abandon all static readings and collect all necessary data by slow speed moving load tests using the dynamic recording equipment. Test runs at the higher speeds desired to evaluate the effects of vehicle speed were made in the same series of runs with each of the gag combinations set up for these slow speed runs. The actual speed of the vehicle

was determined by the timing lines on the oscillograph record. These are not visible on this reproduction but run completely across the record from top to bottom at intervals of 0.01 second, permitting accurate time measurements.

A graphic summary of the results of moving load runs at three different speeds on each of two particular gages is presented in Fig. 21. The oscillograph records in this case have been re-plotted in order to reduce them all to the same length. This permits direct comparison of effects of the load at any given position for each of the three speeds. It will be noted that the slow speed run at 3.5 miles per hour shows very little oscillations. The sharp loops in the trace in each case are the "pips" discussed previously due to the front and rear wheels of the test vehicle passing over the gage location. It will also be noted that these "pips" occur in combination with the oscillations due to vibration in the high speed runs, requiring careful interpretation of these records.

A typical application of the results to the interpretation of structural behavior is illustrated in Fig 22. Here we seek to determine the extent to which composite action between the slab and girder exists at each of two cross sections. At section 19.5, subjected to positive bending moment, the plot of strains across the section shows an essentially linear variation from top to bottom, indicating complete composite action. In the plot of strains for section 21.0, the linearity of strain is in two segments, with a sharp discontinuity at the junction of the slab and girder. Hence, it is obvious that composite action has partially broken down at this section subjected to negative moment.

CONCLUSIONS

1. Investigation of live load effects on full scale bridge structures in the field by using standard instrumentation for static testing presents many great difficulties not encountered in the laboratory unless the number of gages to be read and the number of load positions to be used is small.
2. When available, the use of dynamic type recording equipment to obtain essentially static test data by slowly moving loads offers great advantages in eliminating troublesome temperature effects, detection of localized strain patterns, and in saving time.
3. If it is planned from the start of a test program to employ such continuous recording type equipment, great simplification is possible in the installation of gages, since the importance of long term stability of gages and of temperature compensation of individual circuits is greatly reduced. It is believed that it probably would be possible to use a single dummy gage for large numbers of active gages and to use only single wire leads, grounding the other side of the gage circuit to the structure.

ACKNOWLEDGMENTS

The San Leandro Creek Bridge Project was under the general direction of Professor T. Y. Lin of the Division of Civil Engineering, University of California. Liaison with the Institute of Traffic and Transportation Engineering was provided by Mr. R. Horonjeff. Mr. V. A. Plumb acted as field engineer in charge of loading tests, and later was responsible for the reduction of field data. Acknowledgment for their assistance in the stress measurement program is also made to the members of the Research Advisory Committee and to the many other persons who contributed to the work.

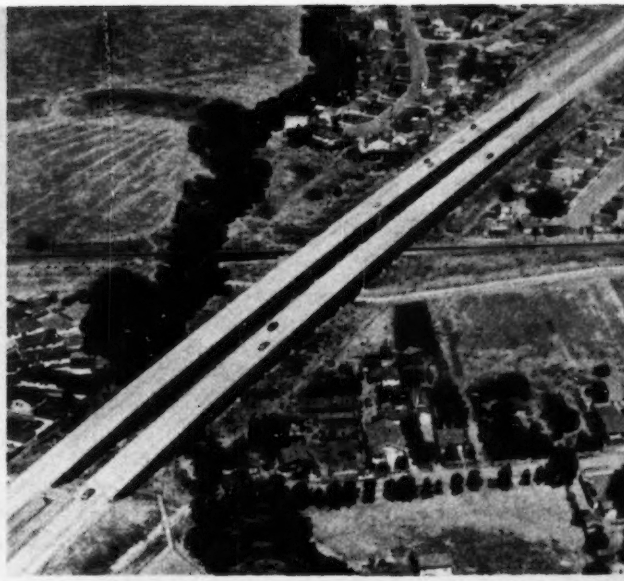


FIG. 1. San Leandro Creek Bridge from the Air.

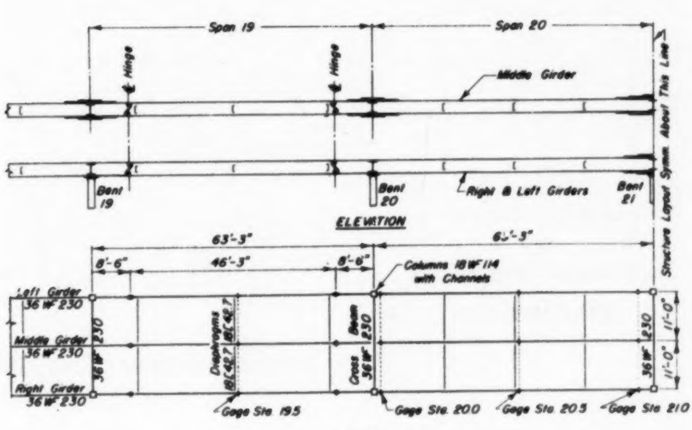


FIG. 2. Steel Layout of Test Spans.

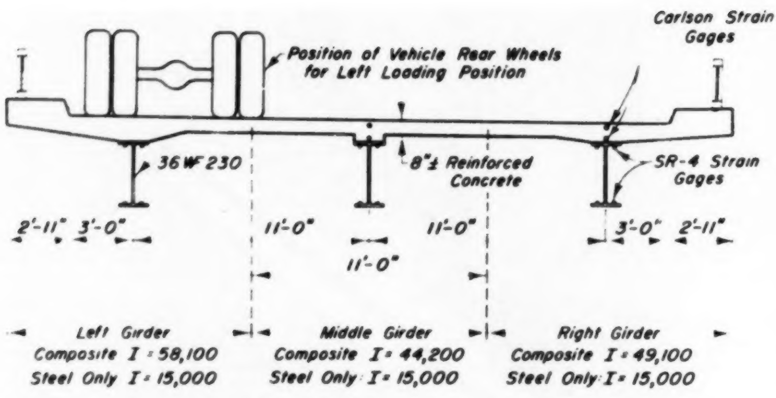


FIG. 3. Cross Section of Bridge, Gage Stations 19.5 and 20.5.

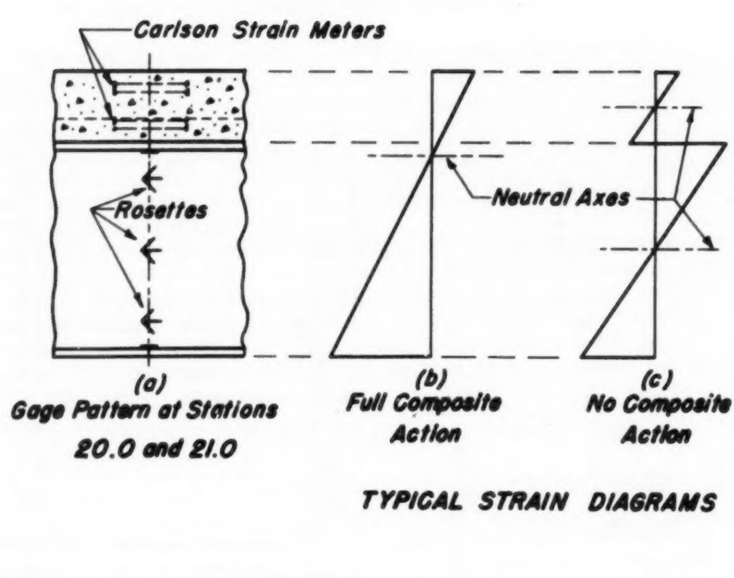


FIG. 4. Arrangement of Gages Adjacent to Support.

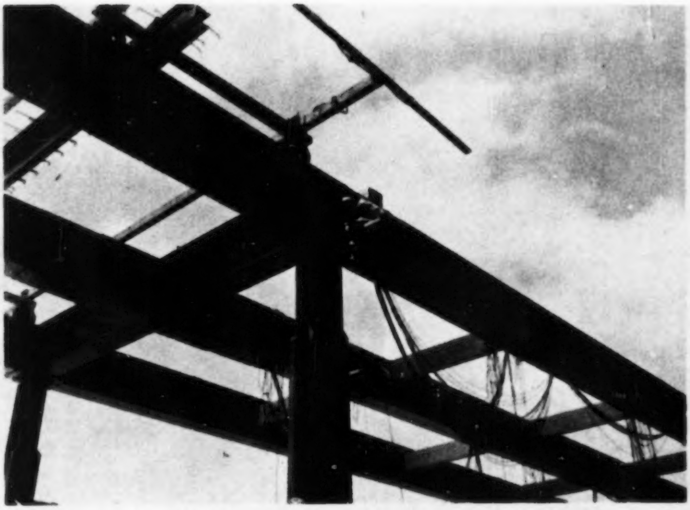


FIG. 5. Installing SR-4 Gages on the Right Girder.

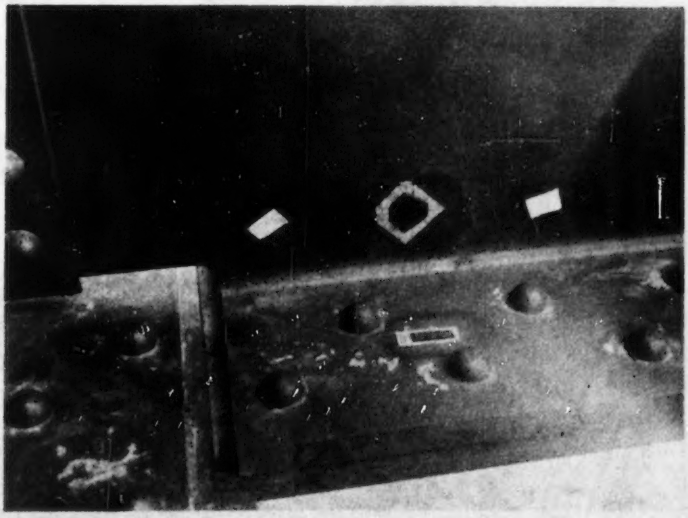


FIG. 6. SR-4 Gages Prior to Wiring and Waterproofing.

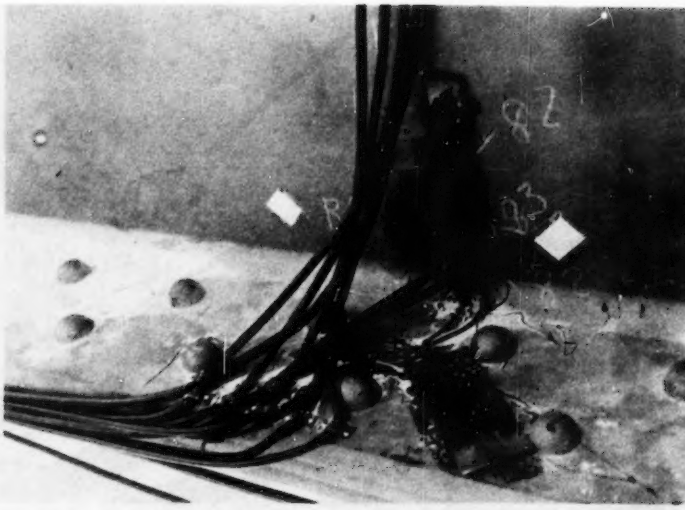


FIG. 7. Active and Dummy SR-4 Gages after Wiring and Waterproofing.

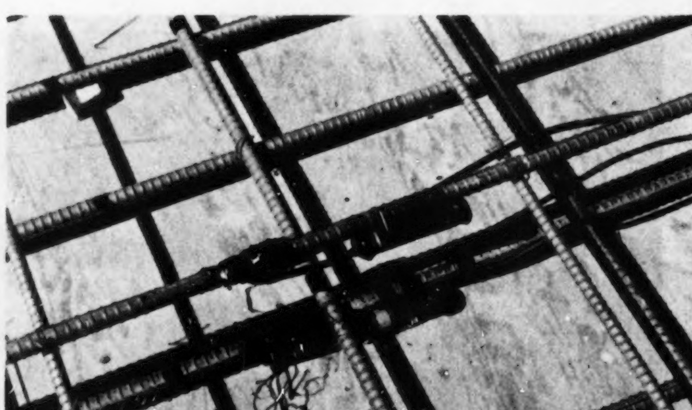


FIG. 8. Active and Dummy SR-4 Gages Installed on Reinforcing Bars.

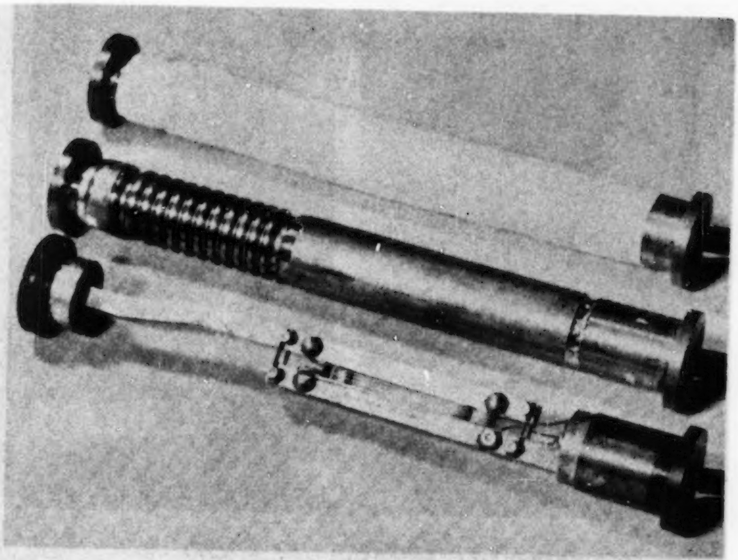


FIG. 9. Carlson Strain Meter.

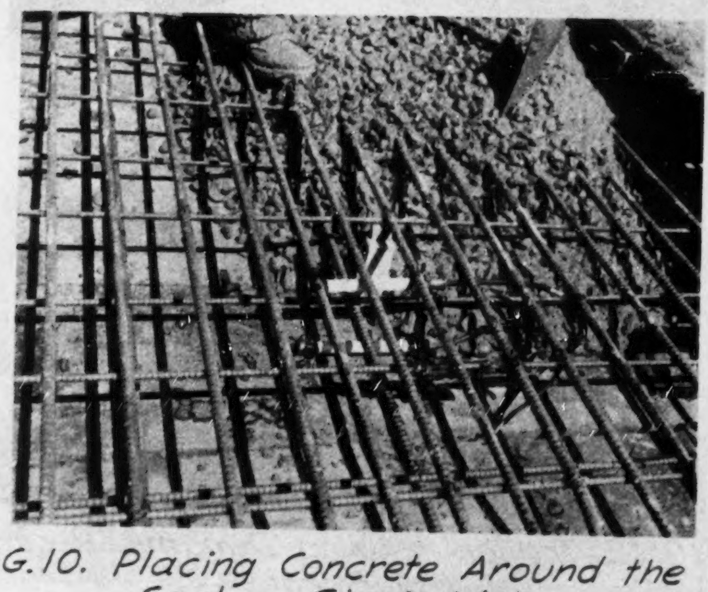


FIG. 10. Placing Concrete Around the Carlson Strain Meters.

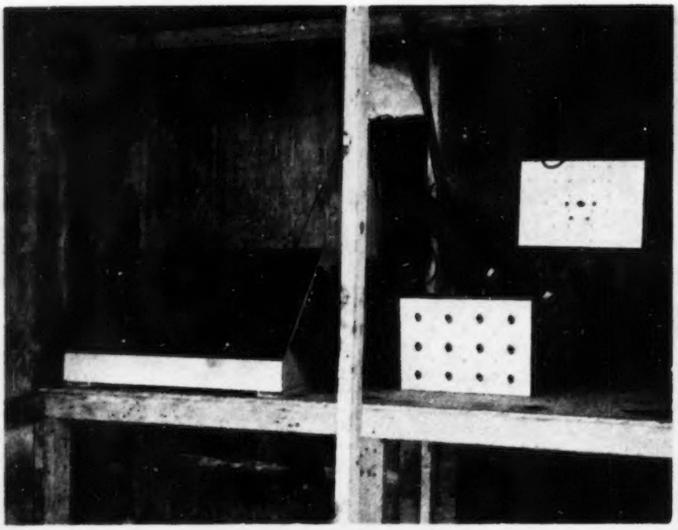


FIG. 11. Switch Boxes in Instrument Shack.

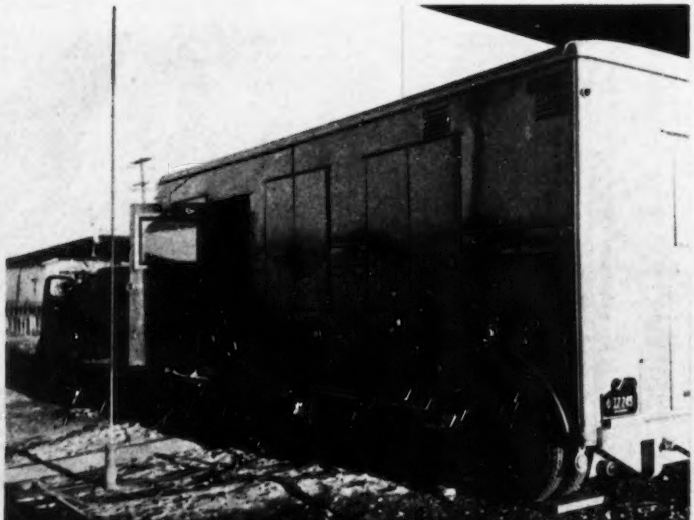


FIG. 12. Trailer Housing Dynamic Recording Equipment.

FIG. 13. Dynamic Recording Equipment in Operation.

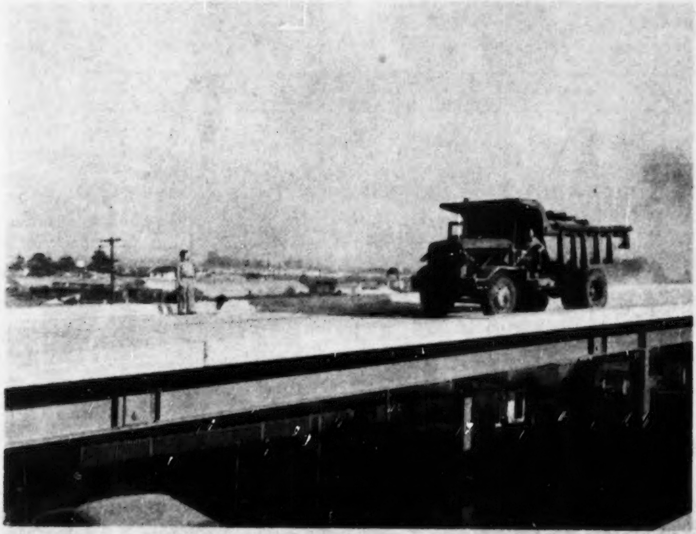


FIG. 14. Test Vehicle of 67000 pounds on Completed Bridge.

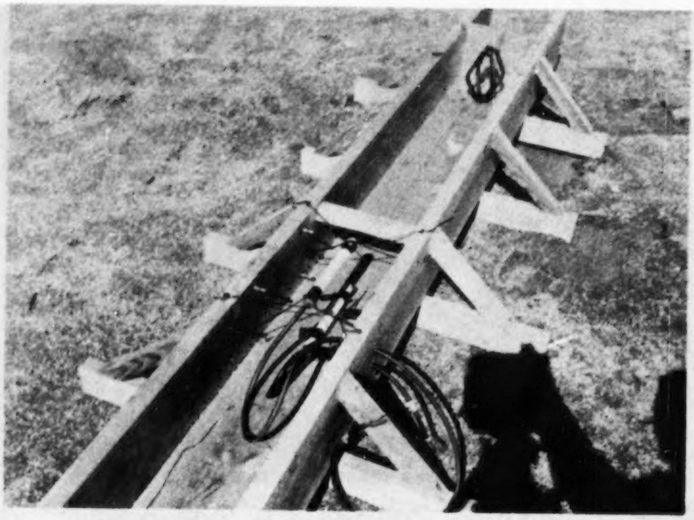


FIG. 15. Form for Concrete Flexure Test Beam.

FIG. 16. Determining Flexural Modulus of Elasticity of Concrete Beam.

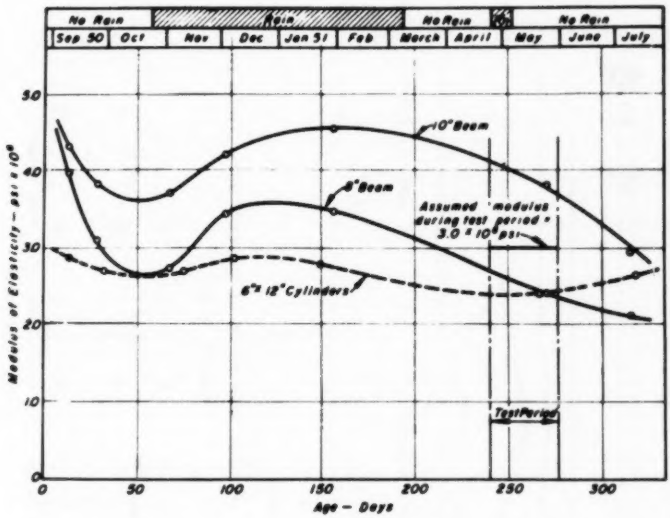


FIG. 17. Modulus of Elasticity of Concrete.

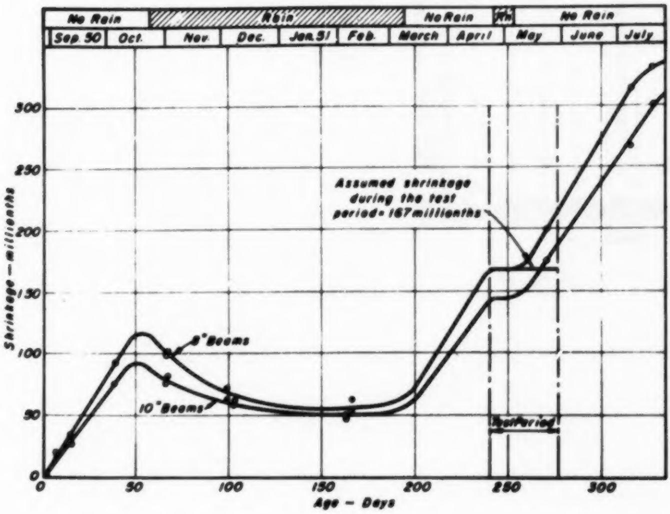


FIG. 18. Shrinkage of Concrete.

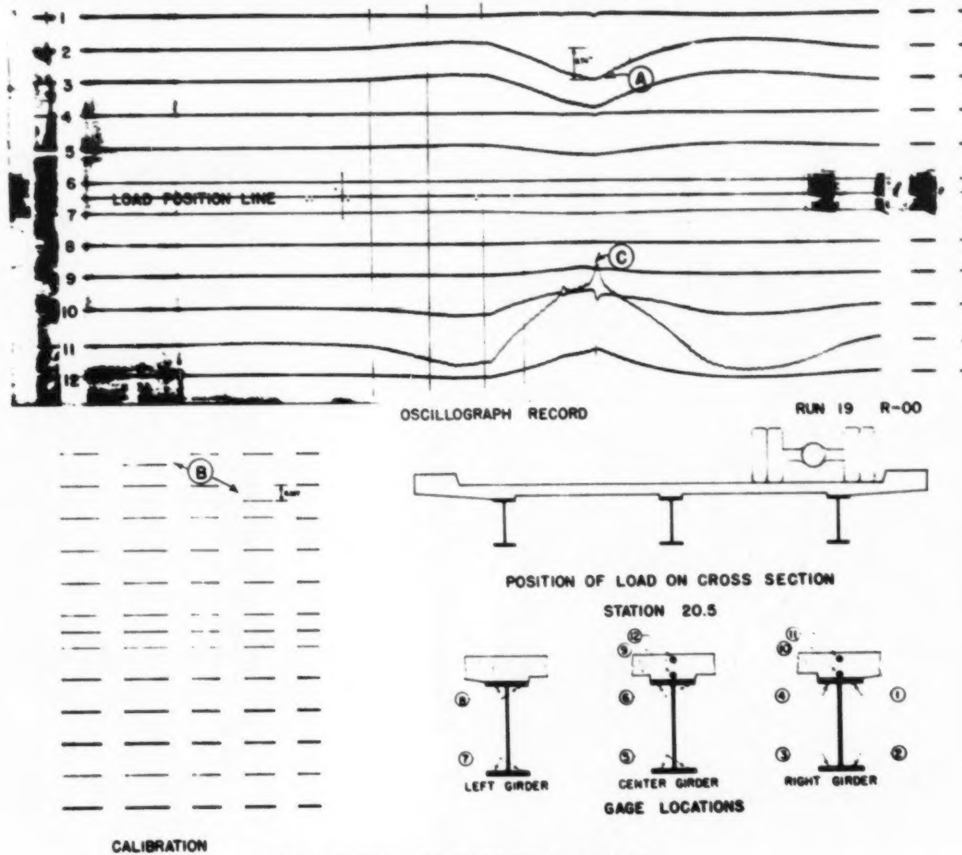


FIG. 19 TYPICAL OSCILLOGRAPHIC RESULTS

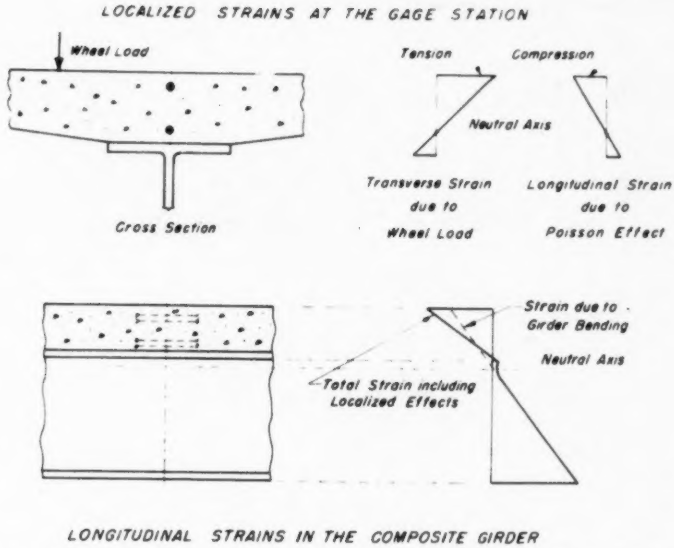


FIG. 20. Effect of Localized Strains.

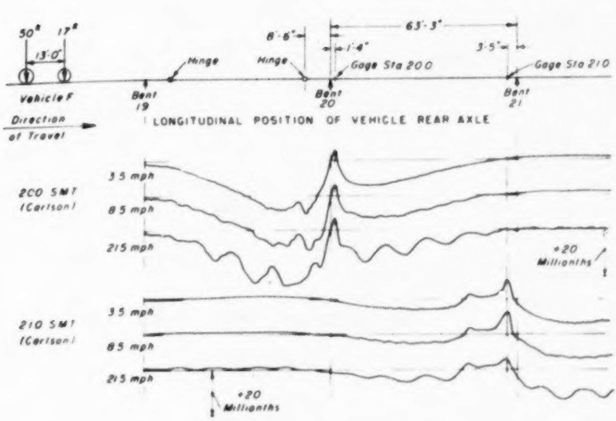
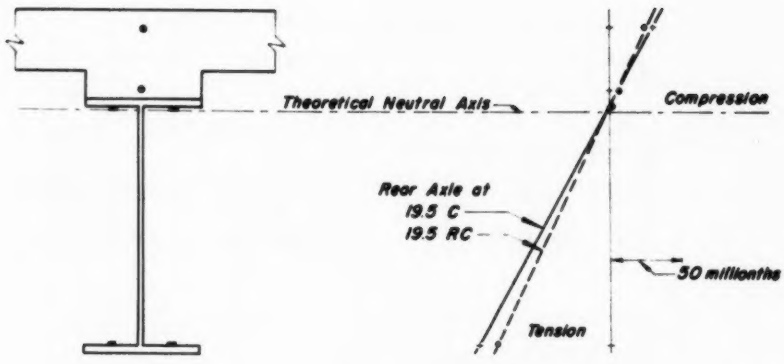
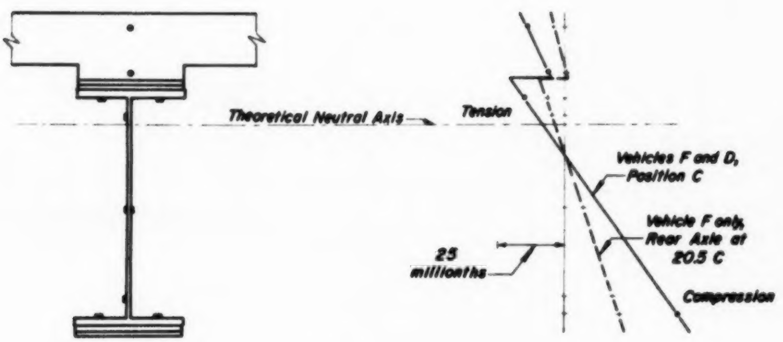


FIG. 21. Typical Dynamic Records.



GAGE STATION 19.5-MIDDLE GIRDER



GAGE STATION 21.0-MIDDLE GIRDER

FIG. 22. Interpretation of Strain Cross Sections.